



Seismic resonance behavior of soil-pile-structure systems with scour effects: Shake-table tests and numerical analyses

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ABSTRACT

The seismic resonance behavior of soil-pile-structure systems (SPSS) with scour effects is a complex problem due to the interaction of soil, pile foundation, and structure. This study performs a series of shake-table tests to identify resonance phenomena of scoured SPSS, which is represented by a simplified bridge model comprised of a lumped mass as the superstructure, a single column, and a cap supported by a 2×2 pile group embedded into uniform sand profiles with different scour depths. Besides, parametric analyses are conducted based on a validated numerical model to quantify the resonance periods for SPSS with different scour depths. The numerical model is characterized by a multi-dimensional soil-pile-structure coupled finite element model with a rectified soil material constitutive model to account for low effective stress effects at shallow depths. It is found that the resonance period of SPSS generally increases with increasing scour depth and excitation intensity, mainly due to the structural nonlinearity of SPSS. Quantitatively, a resonance period between 1 and 2 times the SPSS elastic fundamental period is witnessed for the examined SPSS cases. This study can be a preliminary document for the advanced seismic design of SPSS to mitigate the potential resonance behavior of SPSS with scour effects.

1. Introduction

The general definition of resonance is to describe the phenomenon of increased amplitude that occurs when the frequency of a periodically applied force is equal or close to a natural frequency of the system on which it acts. In earthquake engineering, the term *resonance* describes a circumstance that responses of structures to dynamic loads are significantly amplified if frequency contents of the dynamic loads approach fundamental frequencies of the structures (Clough and Penzien, 2003). Such an excitation frequency (or period) that triggers the maximum response is the so-called *resonance frequency (or period)*. For a single-degree-of-freedom structure, the resonance period is exactly equal to the fundamental period of the structure. For a multi-degree-of-freedom system like most infrastructure facilities in our built environment where both structures and soils are involved and interacted, the resonance period depends on not only the fundamental one but also the second-, third-, and even high-order periods of the system; thereby resonance periods are expected beyond the fundamental period, but quantitative results on such resonance periods for complex

soil-structure systems have yet to be well documented.

The seismic resonance behavior of soil-structure systems has always been a complex problem due to the incomprehension of soil-structure interaction mechanisms. In the past 50 years, many attempts have been made to understand mechanisms of soil-structure interaction for buildings, bridges and other structures via profoundly theoretical analyses [e.g. (Lee and Wesley, 1973)], elaborately laboratory experiments [e.g. (Kitada et al., 1999; Tokimatsu et al., 2005; Hokmabadi et al., 2015; Shang et al., 2018)], field measurement [e.g. (Gara et al., 2019)], and sophisticated numerical simulations [e.g. (Mylonakis et al., 1997; Jeremić et al., 2009; Zamani and El Shamy, 2013; Ghandil et al., 2016)]. More specially, many studies focus on the interaction between soils and shallow spread foundations (Pitilakis and Clouteau, 2010; Ha et al., 2014; Torabi and Rayhani, 2014; Isbilioğlu et al., 2015; Olarte et al., 2017). For example, Torabi and Rayhani (2014) found that seismic resonance is prone to occur in tall structures located at sites of thick soil deposits with low shear wave velocities. Isbilioğlu et al. (2015) investigated the effect of soil-structure interaction on the resonance behavior of building clusters, and indicated that such an effect would reduce the

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resonance amplitude for soft soil deposits in particular. By contrast, relatively few studies report resonance problems in deep pile-foundation-supported structures, which are characterized by more complex interplay in the soil-pile-structure systems (SPSS) (Takewaki, 1998; Rovithis et al., 2009; Hokmabadi et al., 2015). Without being exhaustive, it is worth highlighting a pertinent example. Rovithis et al. (2009) found that structural inertial effects are the main contributor for pile-head bending moment of slender/flexible piles when excitation periods are close to the SPSS fundamental periods. They also found that bending moment at deep depth of pile is mainly generated by kinematic effects when higher soil modes are mobilized under seismic excitations with dominant frequency contents out of the structural fundamental one.

For bridges in flooding-as well as earthquake-prone regions, the seismic resonance problem becomes even more complex due to scour effects that not only change dynamic characteristics of SPSS, but also vary the seismic wave propagation in soil deposits (Shang et al., 2018). As flooding and associated scour effects are variable and sometimes unpredictable during life cycles of bridges, understanding the seismic resonance behavior of SPSS with scour effects is of great importance for the seismic design of scoured SPSS. Many studies have investigated the combined effects of scour and earthquake on demand, capacity, and fragility characteristics of bridges (Alipour et al., 2013; Wang et al., 2014, 2015, 2019c; Klinga and Alipour, 2015; Guo et al., 2018; Fioklou and Alipour, 2019; He et al., 2020; Liang et al., 2020; Liu et al., 2020; Song et al., 2020; Zhou et al., 2021), while very few studies reported the seismic resonance behavior of bridges with scour effects where evolving soil-pile-structure interaction is involved during earthquakes. To mention a highly pertinent study, Xu et al. (2016) adopted numerical analyses to identify critical scour depths that lead to amplified demands of bridges with overlarge pile foundations due to the resonance between the seismic input, foundation, and superstructure. To date, insights on the seismic resonance behavior of SPSS with scour effects are far from well documented. More importantly, quantitative solutions are yet to be developed to estimate the adverse resonance periods of seismic excitations that result in significantly amplified demands under scour effects.

To fill in the above gaps, this study performs shake-table tests and associated numerical analyses to identify seismic resonance behavior of SPSS with scour effects. This paper is organized in two main parts. In the first part (Sections 2 and 3), a series of shake table tests are designed and conducted to demonstrate seismic resonance phenomena in SPSS in the presence of scour effects. Dynamic characteristics and resonance behavior in terms of acceleration and curvature responses are interpreted. For safety of the tests as well as simplifying the problem, structural nonlinearity is not involved in the shake-table tests. In the second part (Sections 4 and 5), a refined soil-pile-structure numerical model is built and validated against the shake-table tests. This numerical model considers nonlinear soil and structural constitutive models. It is used for an extensive parametric study to quantify the seismic resonance periods of SPSS with a wide range of scour depths and excitation intensities that can mitigate the limitation of structural elasticity in the shake-table tests. Conclusions and limitations are addressed finally. The findings of this study can enhance the understanding of seismic resonance behavior and the design of scoured bridges.

2. Shake-table test preparation

2.1. Background of structural prototype and test facilities

The SPSS prototype represents a typical bent for the widespread multi-span reinforced concrete (RC) box-girder bridges in eastern China. The bent consists of a single column (2.2 m in diameter and 10 m in length) and a 2×2 pile-group (1 m in diameter and 19 m in length), which is cast-in-place bored and embedded into medium-dense sandy deposits with the potential of scour.

A laminar box was utilized to prepare an SPSS test model that is

detailed in the next section. The laminar box has an interior dimension of 2 m (longitudinal, i.e., excitation direction) $\times$ 1.5 m (transverse) $\times$ 2 m (vertical). The boundary effect of soil box can be generally negligible at distances over 0.4 m from the box wall (Wu et al., 2002). The soil box was mounted on a 4×6 m shake table located at the Earthquake Engineering Hall of Tongji University. The shake table has a vertical carrying capacity of 70 tons and a horizontal acceleration capacity of 1.5 g. The available input frequency of the shake table ranges from 1 to 50 Hz. The stability of this shake table under a series of sequential excitations has been verified (Wang et al., 2020a). For legibility, the prepared shake-table test model with construction process is displayed in Fig. 1 and further detailed in the following sections. Note that although this figure shows multiple specimens under construction (Fig. 1(b)), the present study only focuses on one specimen against different scour effects, while other specimens were used for other research purposes that are out of the scope of the present study (Shang et al., 2018; Wang et al., 2019c, 2020a).

2.2. Development of SPSS test model

Given the dimension limitation of the laminar box as well as the carrying capacity of the shake table, a scaling-factor of 10 in length was used to scale down the prototype. Accordingly, as shown in Fig. 2, the pile diameter was scaled to $D = 0.1$ m, while the column diameter was scaled to 0.214 m, rather than 0.22 m, to fit for the availability of circular shuttering for the ease of concrete filling. According to the well-known similitude laws proposed by Iai et al. (2005), Table 1 lists the adopted scaling-factors for the 1 g shake-table tests in this study. For the convenience of shake-table testing description and interpretation, dimensional variables mentioned in the following contents are in the model scale unless otherwise specified.

This study concentrates on general scour scenarios. According to previous studies on flood return periods and associated scour depth quantification (Briaud et al., 2014; Yilmaz et al., 2018; Pizarro and Tubaldi, 2019; Pandikkadavath et al., 2022), three cases with scour depths of $SD = 0D$, $2D$ and $4D$ were considered in sequence, i.e., by removing the top $2D$ -thickness soil layer in turn.

According to the length-scaling-factor of 10, perfectly following the similitude law inevitably leads to very small diameters of rebars in pile and column sections. A common choice for model construction is steel wires. Steel wires, however, are too thin to attach strain gauges for monitoring strain and curvature responses of RC pile and column sections. In this regard, section reinforcements were re-designed using steel rebars that are thick enough to attach strain gauges. A longitudinal reinforcement ratio of 2% and a transverse reinforcement volume ratio of 0.8% were adopted to represent the prototype as well as to avoid rebar buckling failure (Dhakal and Su, 2018). Specifically, the longitudinal reinforcement of the single column was provided by ten $\phi 10$ mm rebars with a concrete cover of 2 cm, while the transverse confinement was achieved by spiral $\phi 6$ mm stirrups with an interval of 6 cm. The longitudinal rebars and dense spiral stirrups were extended into the cap to ensure the fixed column-cap connection. As for the piles, six $\phi 6$ mm rebars were assembled as longitudinal reinforcements with a cover of 1 cm, while $\phi 3.5$ mm spiral stirrups with a 4 cm interval provide the transverse reinforcement.

As fundamental periods of ordinary bridges are often between 0.2 and 1.0 s (AASHTO, 2012), the lumped mass that mimics the prototype superstructure was determined as 4 tons to reach this goal; a fundamental period of 0.4–0.5 s was obtained for the test model. Also, the assigned lumped mass leads to an axial load ratio of 5% that follows the prevalent practice for reinforced bridge column design (Aviram et al., 2008). The top of the single column was cast together with a large steel plate that has screw holes for mounting the lumped mass to mimic a fixed bearing connection, while the bearing itself was not considered for the expediency of model preparation and excitation. The single column was supported by a 2×2 pile group via a $0.6 \times 0.6 \times 0.3$ m RC cap. The

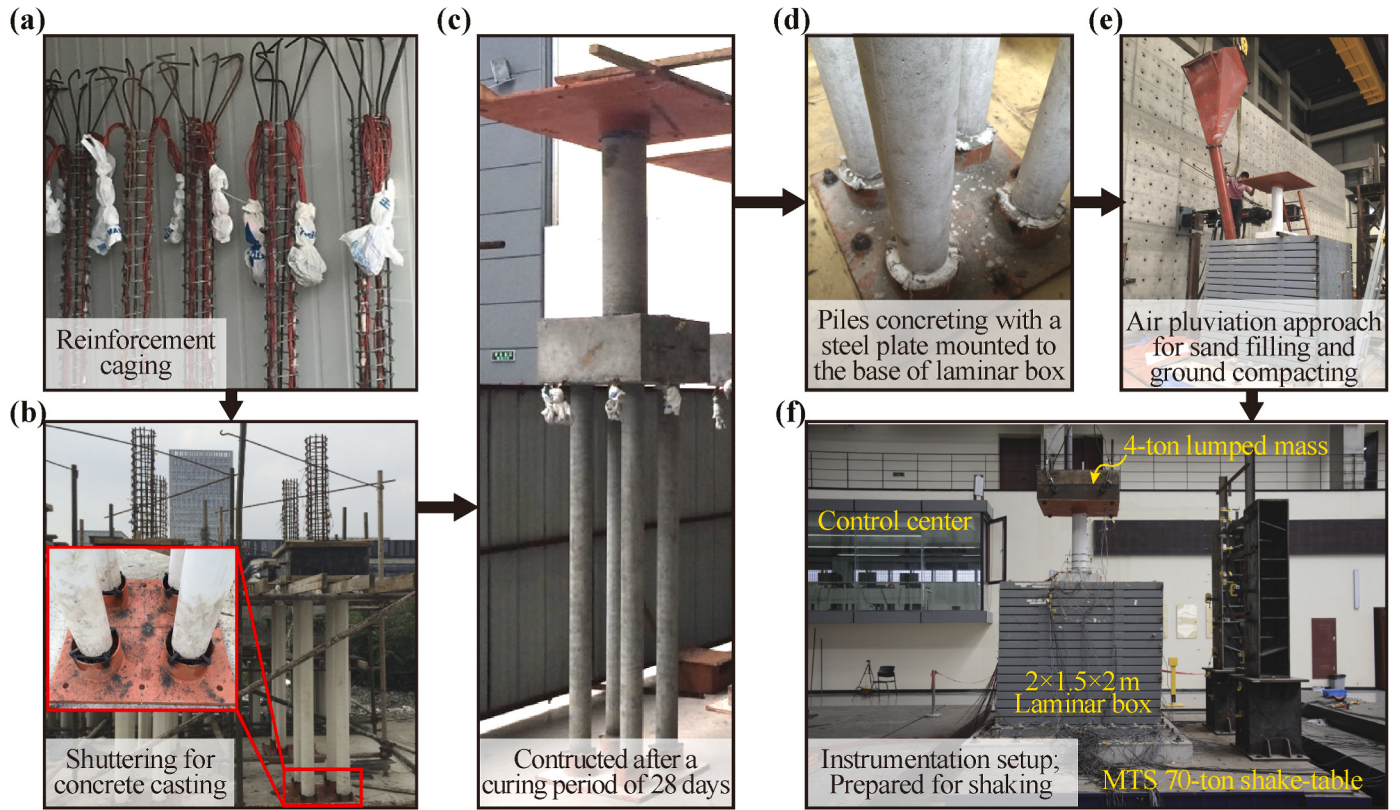


Fig. 1. Photos of the preparation process of shake-table tests: (a) reinforcement caging with installed strain gauges, (b) pile-structure model shuttering for concrete casting, (c) constructed model, (d) model installed at the center of laminar box base, (e) sand ground preparation via the air pluviation approach, and (f) prepared SPSS model for shaking.

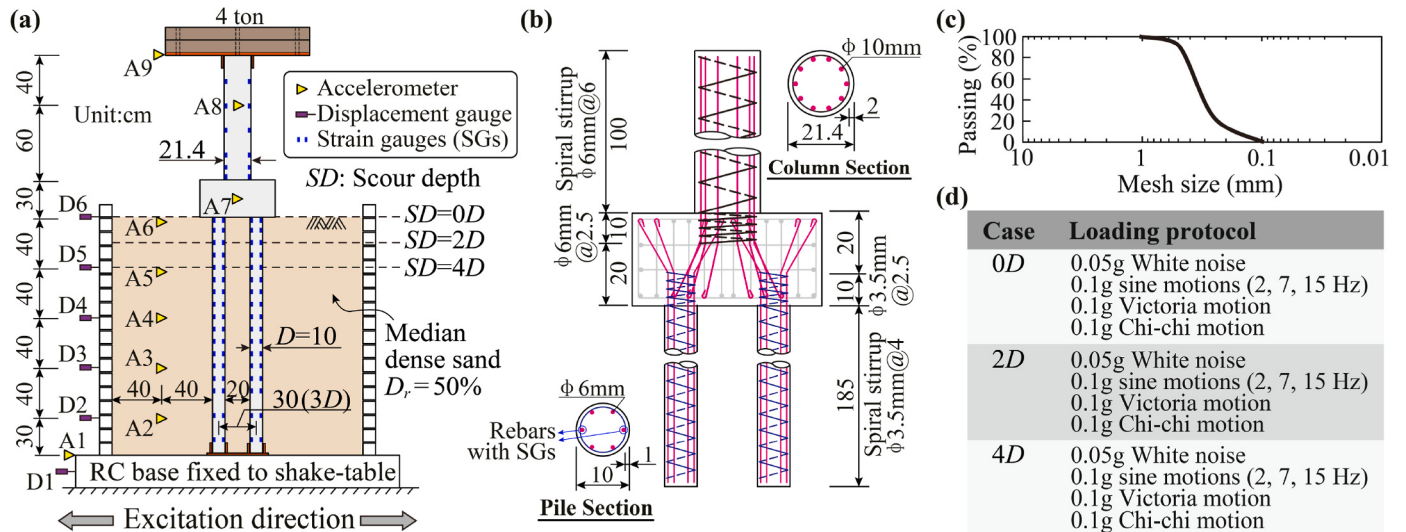


Fig. 2. Details of the SPSS shake-table test model: (a) schematic illustration of instrumentation, (b) reinforcement of RC column and pile, (c) grading curve of sand, and (d) loading protocol.

2 × 2 pile group has a pile center-to-center distance of 3D that follows the common practice for pile foundations (Fleming et al., 2008). For ease of test model construction and installation as well as for the safety of testing, reinforcement cages at pile-tips were soldered with collars of a steel plate, each using two crossed $\phi 6$ mm rebars (see Fig. 1(b)) before concreting them together. The steel plate was mounted to the base of laminar box to simulate a condition of pile foundation embedded in a medium to firm stratum on bedrock. Apparently, such a vertical

boundary condition at pile-tips is not ideally fixed, which will be represented by an elastic spring with a calibrated stiffness in the later numerical simulation.

Shanghai local sands were used for preparing a uniform soil profile with a relative density, $D_r = 50\%$. The used sand is poorly graded with a mean grain size of 0.33 mm, a coefficient of uniformity of 2.06, an internal frictional angle of $\varphi = 31^\circ$, and maximum and minimum dry densities of 1.654 and 1.429 g/cm³, respectively. Accordingly, a natural

Table 1
Scaling-factors used for 1 g shake-table tests in this study.

Quantity	Scaling-factors based on similitude laws for 1 g tests (Iai et al., 2005)	Scaling-factors adopted in this study (Prototype/Model)
Length	μ_L	10
Density	μ_ρ	1.0
Strain	μ_ϵ	1.0
Acceleration	1.0	1.0
Frequency	$(\mu_L \cdot \mu_\epsilon)^{-0.5}$	0.32
Time (Period)	$(\mu_L \cdot \mu_\epsilon)^{0.5}$	3.2
Displacement	$\mu_L \cdot \mu_\epsilon$	10
Curvature	$(\mu_L \cdot \mu_\epsilon)^{-1}$	0.1

density of $\rho = 1.533 \text{ ton/m}^3$ leads to the target $D_r = 50\%$. An air pluviation approach was adopted to place the sand into the laminar box layer by layer to achieve the target soil profile, as detailed in the former studies of the authors (Wang et al., 2018b, 2019c). Note that only dry sand was considered in the present study to avoid liquefaction (Wang et al., 2023).

2.3. Instrumentation

The laminar box and test model were instrumented with a series of accelerometers and strain gauges. As shown in Fig. 2, soil accelerations were recorded at 0, 0.3, 0.7, 1.1, 1.5 and 1.9 m to the base of laminar box (i.e., A1 to A6). Accelerations at the pile-cap, column, and superstructure (i.e., lumped mass) were also measured. Forty pairs of strain gauges were glued on two longitudinal rebars of the column and two of the four piles to monitor the variation of section curvature. The strain gauges on the piles were arranged every 0.1 m (1D), while on the column, they were placed at 0, 0.1, 0.2, 0.4, 0.6 and 0.8 m from the column bottom.

2.4. Excitation protocol for triggering resonance phenomena

The unidirectional excitation protocol for each scour case is listed in Fig. 2(d). The 0.05 g white noise was input first to investigate dynamic characteristics of the SPSS test model. Accordingly, frequencies of sinusoidal waves (2, 7, and 15 Hz detailed later) were determined to trigger resonance phenomena on structural responses. The sinusoidal waves are all in a duration of 25 s, against which the soil and structural peak responses are almost the same for individual cycles; thereby for the convenience of data postprocessing, the peak absolute response at the middle portion of the time history (i.e., 10 s–15 s) are taken as the response. Besides, two scaled in-site ground motions, one from the 1980 Victoria, Mexico earthquake and the other from the 1999 Chi-chi, Taiwan earthquake, were input after the sinusoidal waves for each case. These ground motions are used for numerical model validation later in this study, because in-site ground motions that in nature contain various sinusoidal waves with different frequencies (from a view of Fourier transform) can better validate the numerical model compared with sinusoidal waves with specific frequencies. Note that all the sinusoidal and ground motions were scaled to have a peak acceleration of 0.1 g to assure the test structural model remained elastic for testing safety as well as for the sake of valid comparisons across different excitation and scour conditions, while nonlinear behavior of structures is considered in numerical analyses later in this paper.

3. Interpretation of test results

3.1. Dynamic characteristics of SPSS test model

White noise-induced acceleration time histories at top layer of soil, pile-cap and superstructure are adopted to characterize the dynamic behavior of the three components and their interactions in the SPSS test

model. For legibility, acceleration response spectra (ARS) of the recorded time histories before scour (i.e., $SD = 0D$) are generated and shown in Fig. 3. The test model exhibits mainly three vibration modes: The first mode with a period of $T_1 = 0.40 \text{ s}$ indicates the translational vibration of the superstructure; the second mode shows a period of $T_2 = 0.14 \text{ s}$, representing the vibration of soil layers together with the mobilized motion of the pile-cap; and the third mode (a period of $T_3 = 0.03 \text{ s}$) is also triggered by the movement of soil and pile-cap, as implied by their similar trends near T_3 (see Fig. 3). A numerical study later in this paper will illustrate vibration shapes of the SPSS test model to further address the above interpretation. Note that the three vertical dash-dot lines marked in Fig. 3 indicates the adopted frequencies of sinusoidal waves, as detailed later in this section. Recalling the adopted scaling-factor of 3.2 in period (recall Table 1), the observed first mode period of $T_1 = 0.40 \text{ s}$ for the test model corresponds to a natural period of 1.28 s for the prototype bridge structure. Such a natural period (1.28 s) falls into the common range of natural periods for multi-span RC girder bridges (Wang et al., 2015; Fioklou and Alipour, 2019).

To understand the impact of scour depth on vibration periods of the SPSS model, Fig. 4 shows ARS of the white noise-induced acceleration demands at superstructure, soil top layer, and pile-cap under different SD s (i.e., $SD = 0D, 2D$ and $4D$). From a quick view, scour indeed changes the dynamic characteristics of the SPSS test model due to the removal of soil layers. More specifically, from Fig. 4(a), the fundamental period (T_1) changes from 0.40 s before scour ($SD = 0D$) to 0.45 s under $SD = 2D$ and further to 0.48 s under $SD = 4D$, elongated by as large as 20%, indicating a more and more flexible structure. From Fig. 4(b), by contrast, the period of the soil-dominated second mode (T_2) decreases after scour due to the removal of soil layers that shortens the height of the soil column and thereby increases its stiffness. Recall Fig. 4(a), there is a noticeable second peak around the period of around 0.1 s for each scour condition; the corresponding period apparently becomes larger with the increasing SD . This is also a consequence of scour that reduces constraints to the foundation and accordingly increases the flexibility of the supported structure. On the other hand, the response at pile-cap is more complex because of the coupled contribution of kinematic loads from soil movement and inertial loads from superstructure vibration. In other words, the scour-induced period variation of the pile-cap may be partially affected by the increased period in superstructure, together with the decreased one in soil. As a result, seen in Fig. 4(c), the pile-cap-dominated period decreases from $SD = 0D$ to $2D$ while increases from $SD = 2D$ to $4D$. It should be noted that this interpretation is a preliminarily qualitative explanation, more experimental studies are warranted to identify other sources for justifying the variations of periods from a quantitative viewpoint.

According to the identified primary vibration periods of the SPSS test model under different scour depths, three frequencies namely 2 Hz (i.e., period of 0.5 s), 7 Hz (0.14 s), and 15 Hz (0.06 s) are selected for the later sinusoidal wave excitations. Note that the former two frequencies (2 and 7 Hz that are close to T_1 and T_2 , respectively) are expected to trigger superstructure and soil resonance responses, whereas the third one (15 Hz), which is relatively apart from T_1 and T_2 (see Fig. 3) and thereby may not trigger resonance responses, serves as a foil to highlight the resonance responses at superstructure and soil top layer under the 2 and 7 Hz sinusoidal excitations. On the other hand, as the 15 Hz input is relatively close to T_3 , opportunities exist in using this input to show the resonance behavior between the input and pile-cap.

3.2. Acceleration resonance responses

Soil resonance responses of the SPSS test model under sinusoidal waves are shown in Fig. 5 and 6 in terms of acceleration ratio, which is defined as the peak absolute acceleration response at a position divided by that at the base of the laminar box (i.e., from Accelerometer A1). Fig. 5 focuses on the $SD = 0D$ case to elaborate the role of excitation frequency in the acceleration responses, while Fig. 6 shows a

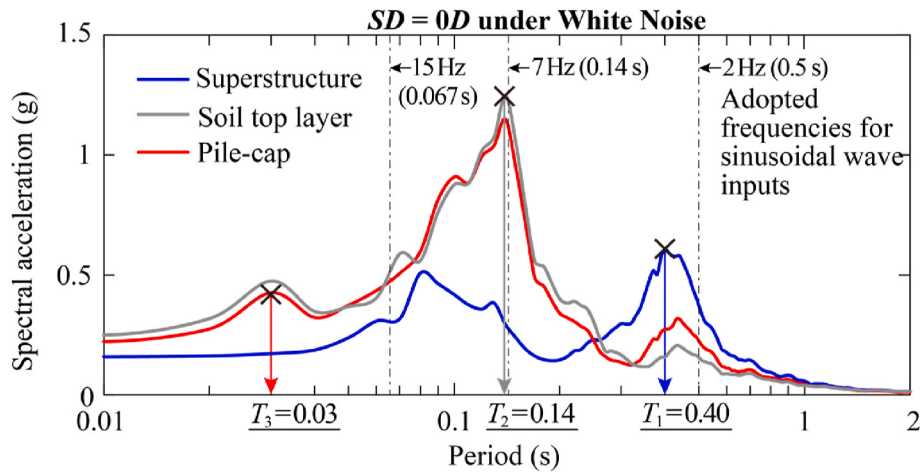


Fig. 3. Acceleration response spectra of the superstructure, pile-cap, and soil top layer of the test model before scour ($SD = 0D$) under white noise excitations.

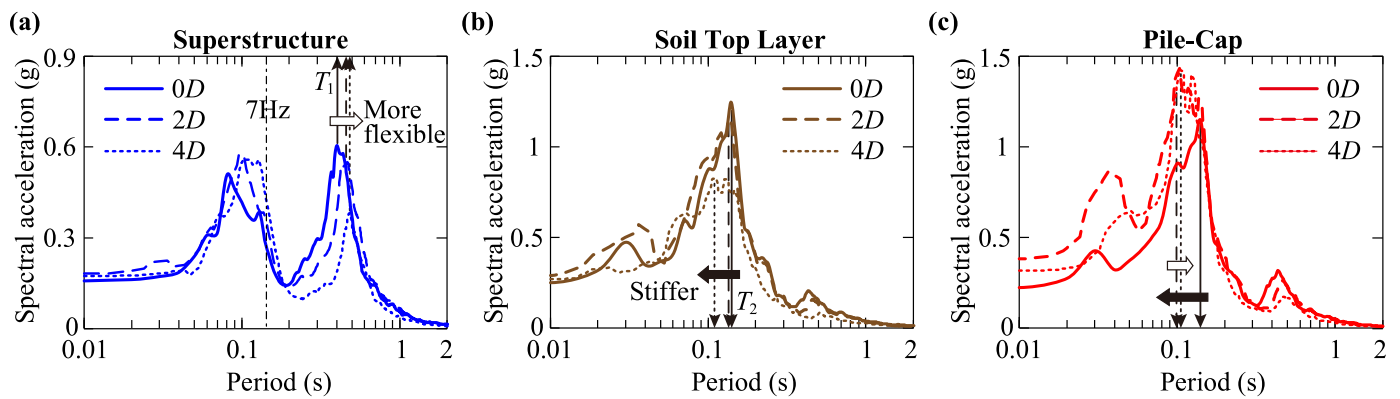


Fig. 4. Influence of scour depth ($SD = 0D, 2D$ and $4D$) on acceleration response spectra under white noises at different SPSS components: (a) superstructure, (b) soil top layer, and (c) pile-cap.

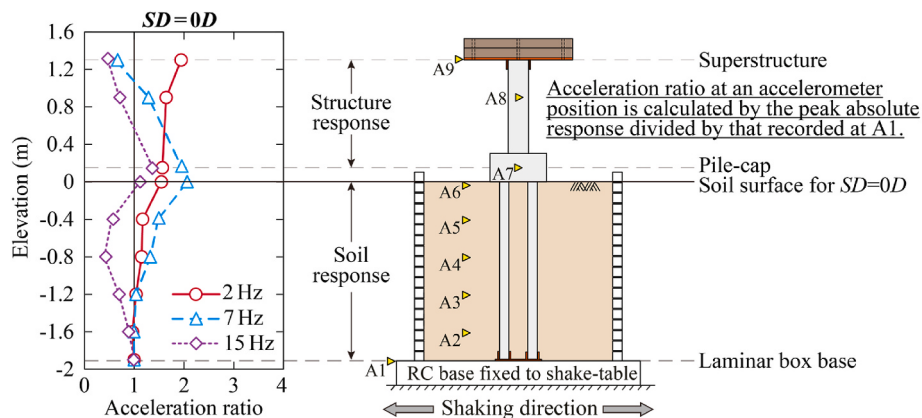


Fig. 5. Influence of sinusoidal excitation frequencies (2, 7, and 15 Hz) on acceleration ratio distributions of the SPSS test model before scour ($SD = 0D$).

comprehensive comparison across all the examined SD s and excitation frequencies. For legibility, acceleration ratios at the pile-cap, column and superstructure are also shown in these figures to better describe the acceleration propagation in SPSS. An acceleration ratio larger than 1.0 indicates an amplification, otherwise it means a degradation of intensity during the propagation.

From a quick view of Fig. 5, the SPSS test model exhibits quite different dynamic behavior across different sinusoidal inputs. In general, the 2 and 7 Hz cases trigger a larger amplification than 15 Hz. More

specifically, for 2 Hz (i.e., a period of 0.5 s that is close to T_1), peak accelerations across the SPSS test model are all amplified due to the resonance of the input and the first mode (vibration of superstructure); the acceleration ratio at superstructure is as large as 2 in particular, and the peak accelerations at pile-cap and soil surface are amplified by a factor of over 1.5. Under the 7 Hz excitation (i.e., 0.14 s, equal to T_2 dominated by soils), the resonance phenomenon is again observed; accelerations at soil surface and pile-cap are amplified by a factor of about 2 while that at superstructure is reduced to almost a half of the base

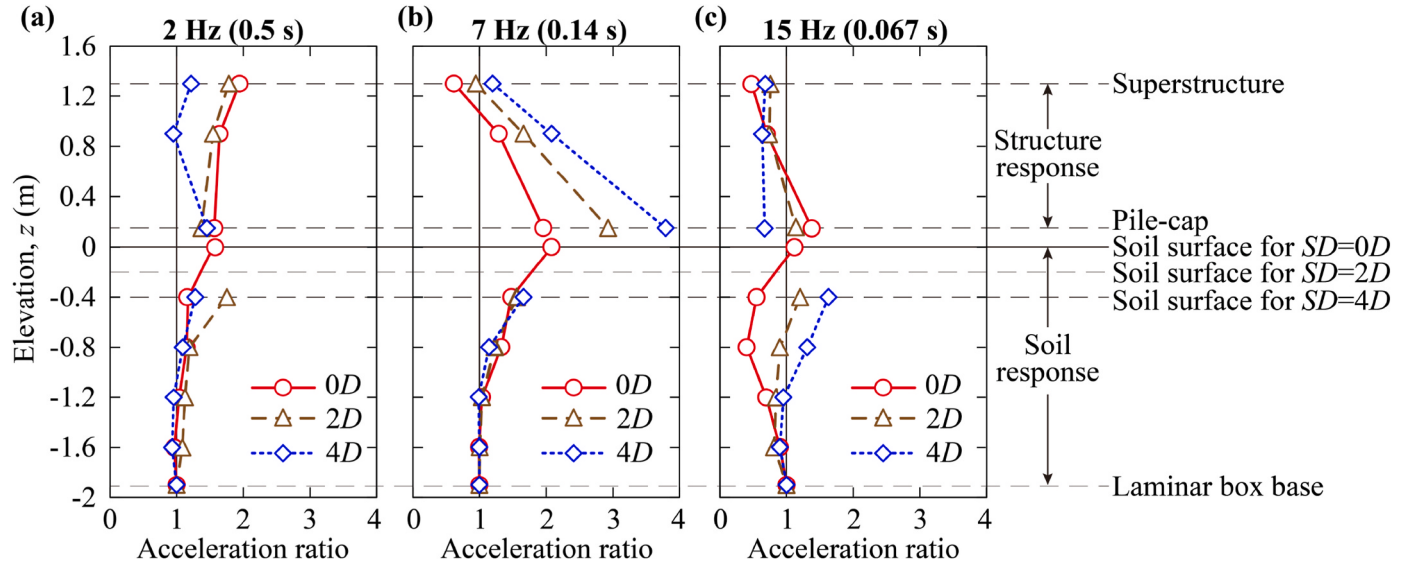


Fig. 6. Influence of scour depth ($SD = 0D, 2D$ and $4D$) on acceleration ratio distributions of the SPSS test model under sinusoidal waves with different frequencies: (a) 2 Hz, (b) 7 Hz, and (c) 15 Hz.

input. As for 15 Hz (0.067s), which is far from T_1 and T_2 (see Fig. 3), but relatively close to T_3 (mainly related to vibrations of pile-cap and soil surface), acceleration responses at the superstructure and most soil layers are apparently reduced, while that at the pile-cap is still amplified by a ratio of approximately 1.3.

From Fig. 6, the role of SD in acceleration propagation is quite different for different excitation frequencies. For 2 Hz (0.5 s) that is close to T_1 , increasing SD from 0D to 2D slightly reduces the column and superstructure acceleration responses. However, when significant scour occurs, e.g., $SD = 4D$, the column and superstructure responses are apparently de-amplified because of the apparently increased structural flexibility; the acceleration ratio decreases from 2.0 to nearly 1.0. By contrast for 7 Hz, which is close to T_2 , the pile-cap and soil accelerations are found obviously larger than the other two cases. More specifically, from Fig. 6(b), the pile-cap responses increase with the increasing SD . This can be explained by the variations of fundamental periods and spectral accelerations shown above in Fig. 4(a) and (c). First of all, the cap acceleration response should be affected by the fundamental periods of pile-cap as well as superstructure, because the first mode of SPSS (i.e.,

superstructure vibration) mobilizes the pile-cap vibration, as can be seen from the peaks at around 0.1 s in Fig. 4(a). Specifically, the pile-cap period of 4D case is closer to 7 Hz, compared with the 2D and 0D cases. Also, the spectral acceleration at 7 Hz is larger in the 4D case than the 2D and 0D cases. From Fig. 4(c), although the fundamental period of 0D case is relatively closer to 7 Hz than the 4D case, the spectral acceleration of 4D case is still larger than the 0D case. Furthermore, it is acknowledged that higher modes usually have smaller modal masses, and thus a lesser impact on the overall response. In other words, the first-mode-mobilized pile-cap period closer to the input, together with the larger spectral acceleration, dominates the overall larger pile-cap acceleration responses at larger scour depths. As for 15 Hz that is far away from T_1 , acceleration ratios at superstructure are always below 1.0 across the examined SD s, while the soil accelerations are amplified because of the increased SD s that stiffen the soil column.

To further characterize the soil acceleration resonance behavior, Fig. 7 takes the $SD = 0D$ case as an example to illustrate relationships for soil responses at different depths versus the shake-table base input with different frequencies (periods). These relationships can be called input-

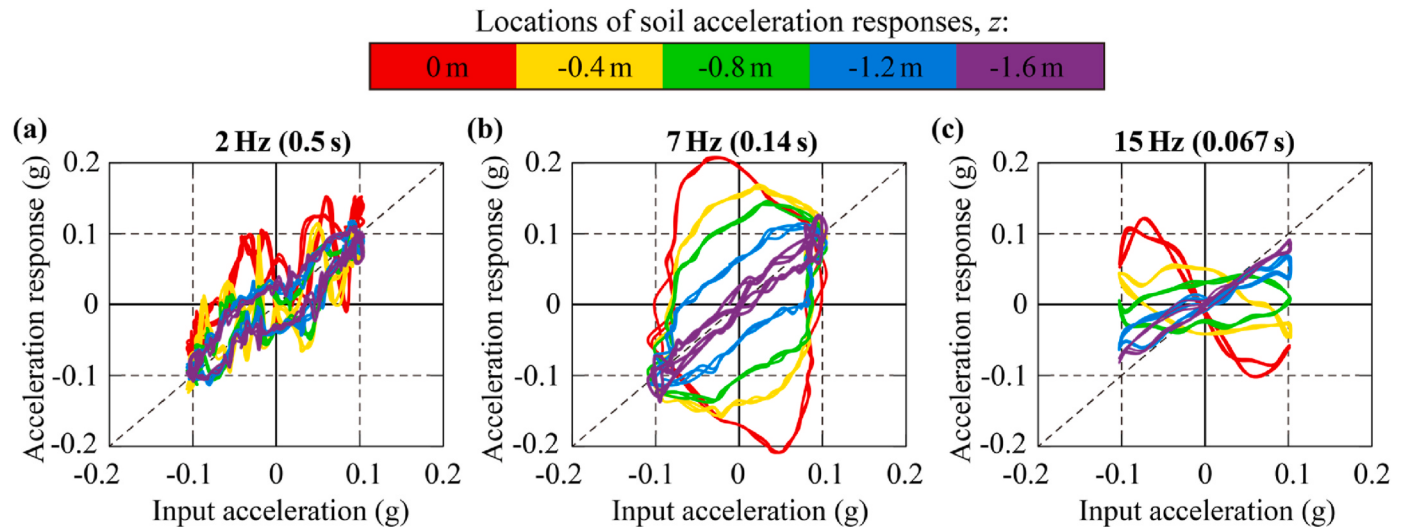


Fig. 7. Relationships between input accelerations and soil acceleration responses at different depths of the SPSS test model before scour ($SD = 0D$) under sinusoidal motions with different frequencies: (a) 2 Hz, (b) 7 Hz, and (c) 15 Hz.

response loops. Note that a loop in the first/third quadrant indicates the soil response and base input accelerations are in the same direction, otherwise they are opposite. From Fig. 7(a) for 2 Hz, the peak acceleration responses across different locations all occur in the first and third quadrants, indicating almost synchronous vibrations between the input and the soils. By contrast, quite different shapes are seen for 7 Hz shown in Fig. 7(b), where the loops change from nearly a line at a deep location ($z = -1.6$ m) to a plump rhomboid at soil surface ($z = 0$ m) due to the significant resonance behavior in soil. Under the 7 Hz and 0.1 g input, the peak acceleration reaches nearly 0.2 g at soil surface, indicating an acceleration ratio of 2.0 as described above in Fig. 5. As for 15 Hz shown in Fig. 7(c), the loop rotates clockwise from the 1:1 line for $z = -1.6$ m to almost horizon for $z = -0.8$ m and finally to nearly a 1:1 line across the second and forth quadrants for the soil surface, indicating an apparent change in phase (i.e., opposite in vibration direction). It is worth noting that the almost horizontal loop at $z = -0.8$ m indicates a minimum acceleration response as reflected above in Fig. 5 for 15 Hz.

Fig. 8 shows acceleration input-response loops for soil surface, pile-cap and superstructure under different sinusoidal excitations for the $SD = 0D$ case as an example. From Fig. 8(a), it is clear the superstructure response is significantly amplified in an opposite direction under 2 Hz excitation due to the resonance behavior between the input and superstructure. From Fig. 8(b), the plump loops at soil surface and pile-cap under 7 Hz imply the significant resonance behavior at these two locations, while the superstructure loops under 7 Hz and 15 Hz are quite close to the horizontal axis, indicating low responses at superstructure, as reflected above in Fig. 5 for these two frequencies.

3.3. Curvature resonance responses

To understand the influence of loading frequencies (2, 7 and 15 Hz) on curvature resonance behavior of the pile and column, Fig. 9 takes $SD = 0D$ as an example to showcase the distribution of curvature at the maximum response epoch. Apparently, curvatures under 2 Hz that corresponds to the SPSS fundamental period (T_1) are much larger than the other frequencies, which indicates that the first mode dominates the structural curvature responses.

Furthermore, Fig. 10 displays curvature distributions for different scour depths under different excitation frequencies. Before interpreting the role of scour depth, it is worth noting the almost zero curvature responses at the pile-tip, which may be forced by the boundary conditions imposed on the system. Under different sinusoidal inputs, the curvature varied significantly on both the magnitude and the distribution shape along the pile height. For 7 Hz and 15 Hz (Fig. 10(b) and (c)),

slight bending behavior is observed in the column due to the small inertial forces on the superstructure, whereas for the pile, both the middle and top portions show obvious bending responses under the 2 Hz excitation (Fig. 10(a)), because the first mode of SPSS (with a fundamental frequency close to 2 Hz) inevitably mobilizes the deformation of pile-cap and piles. Besides, potential failure patterns can be greatly affected by scour depths and excitation frequencies. The distribution shape of curvature under 15 Hz is quite close to that of 2 Hz (the most vulnerable pile portion is both underground), but the magnitude is much smaller due to the obvious difference between the excitation frequency (15 Hz) and the fundamental frequencies of SPSS. For 7 Hz, on the contrary, the maximum curvature occurs on the pile-head, which should be attributed to the closeness of 7 Hz to the fundamental frequency of the second mode of SPSS (i.e., the deflection of soil column mobilizes the larger movement of pile-head than the underground pile).

In summary, the shake-table tests do identify the SPSS resonance behavior under particular excitation periods for superstructure, pile-cap, and soils. However, due to the limited excitation cases and elastic structural behavior in the shake-table tests, quantitative resonance periods under more common nonlinear soil and structural conditions are yet to be well addressed. Thus, FE modeling and associated parametric studies considering soil and structural nonlinearity are performed.

4. Numerical modeling and validation

4.1. Multi-dimensional FE modeling overview

A coupled soil-pile-structure FE model is developed to simulate the shake-table tests, as illustrated in Fig. 11. The FE model is characterized by a multi-dimensional modeling technique to balance the prediction accuracy and computational cost, i.e., two-dimensions (2D) and two degrees-of-freedom (2DOF) in the soil domain, and 3D-6DOF in the structure and soil-pile spring domain. In this manner, the 2×2 pile-group foundation can be completely modeled to capture the seismic behavior of each pile, while the extensive computational burden on traditional 3D soil continua can be released. More specifically, the soil nodes can only move in horizontal (X) and vertical (Y) directions, while the soil slave nodes and structure nodes have six DOF (i.e., an out-of-plane horizontal axis (Z) is assigned in the 3D structure domain to allow these nodes translate and rotate). Note that the soil nodes are tied together with the soil slave nodes at the same depths in the X and Y translational directions using the equalDOF command (Cook et al., 2001) in OpenSees (McKenna et al., 2010), while the rest four DOF of each soil slave node are fixed to effectively transfer the 2D-2DOF soil

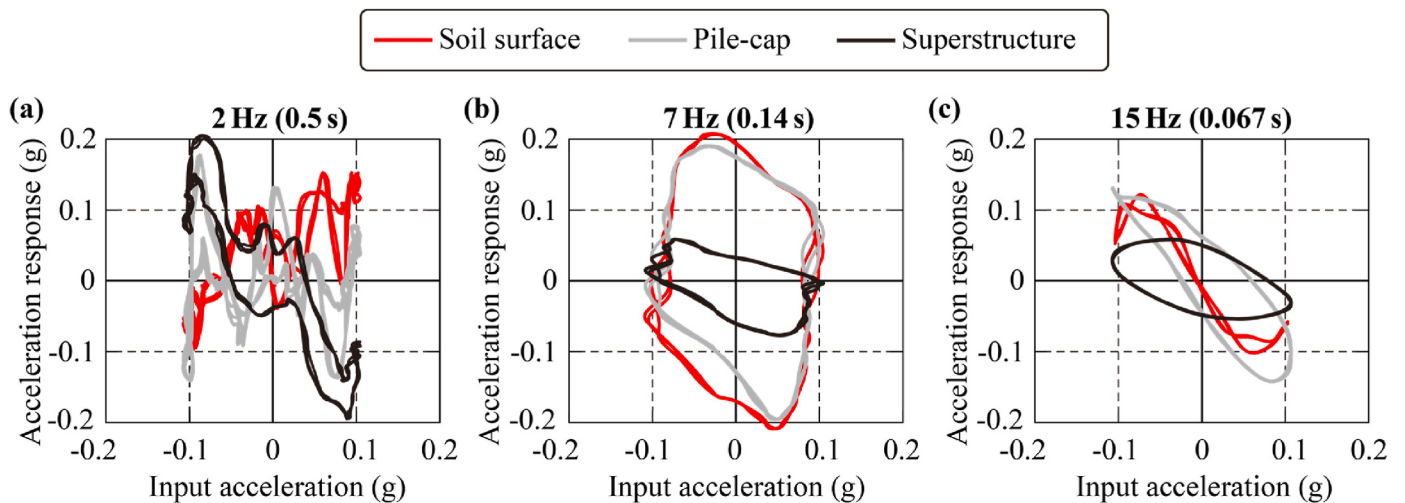


Fig. 8. Relationships between input accelerations and structural responses of the SPSS test model before scour ($SD = 0D$) under sinusoidal motions with different frequencies: (a) 2 Hz, (b) 7 Hz, and (c) 15 Hz.

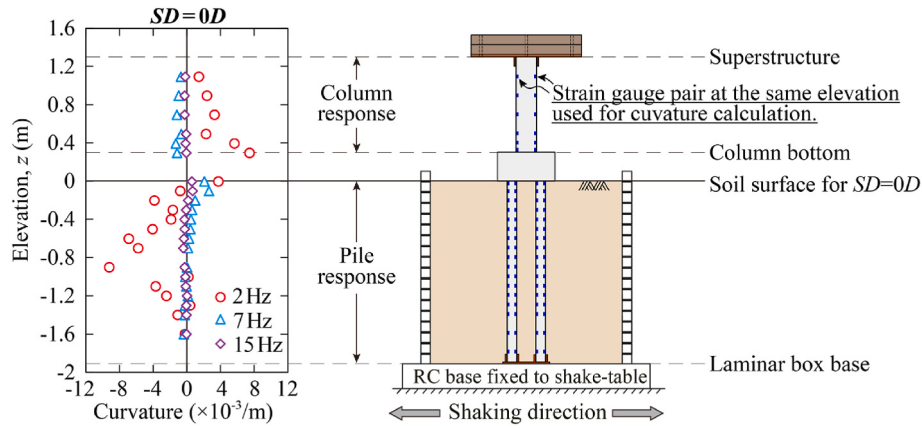


Fig. 9. Influence of sinusoidal excitation frequencies (2, 7 and 15 Hz) on curvature distributions of the SPSS test model at the maximum response epoch before scour ($SD = 0D$).

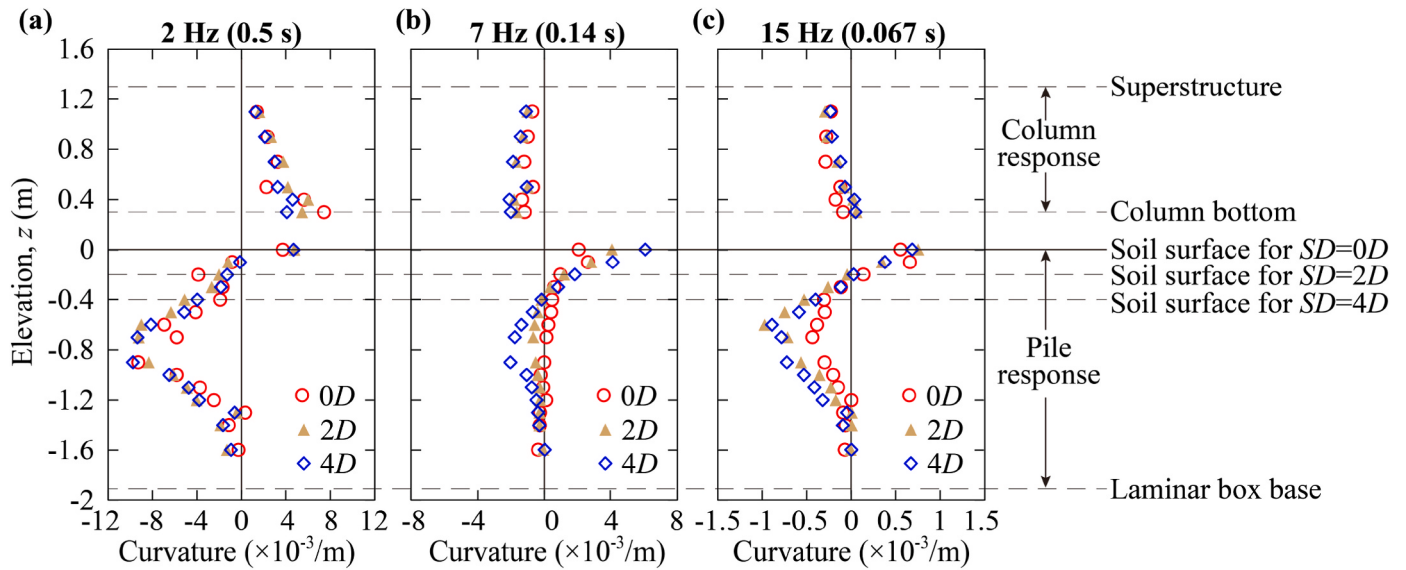


Fig. 10. Influence of scour depth ($SD = 0D$, $2D$ and $4D$) on curvature distributions of the SPSS test model at the maximum response epoch under sinusoidal waves with different frequencies: (a) 2 Hz, (b) 7 Hz, and (c) 15 Hz.

responses to the 3D-6DOF domain. Details of structure, soil, and soil-pile spring modeling are elucidated below.

4.2. Structural modeling

Though piles and column remained elastic in the shake-table test, displacement-based beam-column elements with fiber sections are adopted in the FE model to adapt to the potential nonlinear behavior of SPSS under large earthquakes in the later parametric study. As schematically shown in Fig. 11, the Concrete01 material is used for the concrete cover and core fibers, where constitutive parameters for the concrete core are determined accounting for the confinement from the transverse reinforcement (Filippou et al., 1983). Table 2 lists the adopted concrete model parameters. The longitudinal steel fibers are represented by the Steel02 material together with a minmax material to consider rebar snapping at the ultimate strain, as listed in Table 3. Note that the basic concrete and steel parameters in these two tables were obtained from material mechanics tests just before the shake-table tests. The pile and column elements are meshed by a length of $1D$ (i.e., 0.1 m), each with five integration points. Such an modeling manner results in a reliable prediction of structural curvature responses based on former studies [e.g., (He et al., 2016; Wang et al., 2018a, 2019b; Blanco et al.,

2019), among others]. Besides, elastic beam-column elements are utilized to model the rigid pile-cap and connect the superstructure centroid to the top of the column.

4.3. Soil modeling with modified constitutive parameters accounting for shallow-layer low pressure

The sand ground is represented by plain-strain four-node Quad elements with the pressure-dependent multi-yield surface (PDMY) material (Yang, 2000). Each pair of soil nodes at the same depth is tied together using the equalDOF command to mimic the pure shear behavior of the soil column as the laminar box shook during the test. Gravity effects of the soil elements are represented by vertical body forces. The two nodes at the bottom of soil column are fixed to input the seismic excitation. In-plane and out-of-plane widths of the plain-strain elements follow the prepared soil profiles, while the thickness of each soil element is set to $1D$ (i.e., 0.1 m, equal to the length of pile element), so that the soil column and piles can be linked properly using soil-pile springs as detailed in the next section. Also, such a mesh of the thickness assures that frequencies of interest for the input motions can be reasonably propagated (Zhang et al., 2008).

In the FE model validation process, parameters in the PDMY mate-

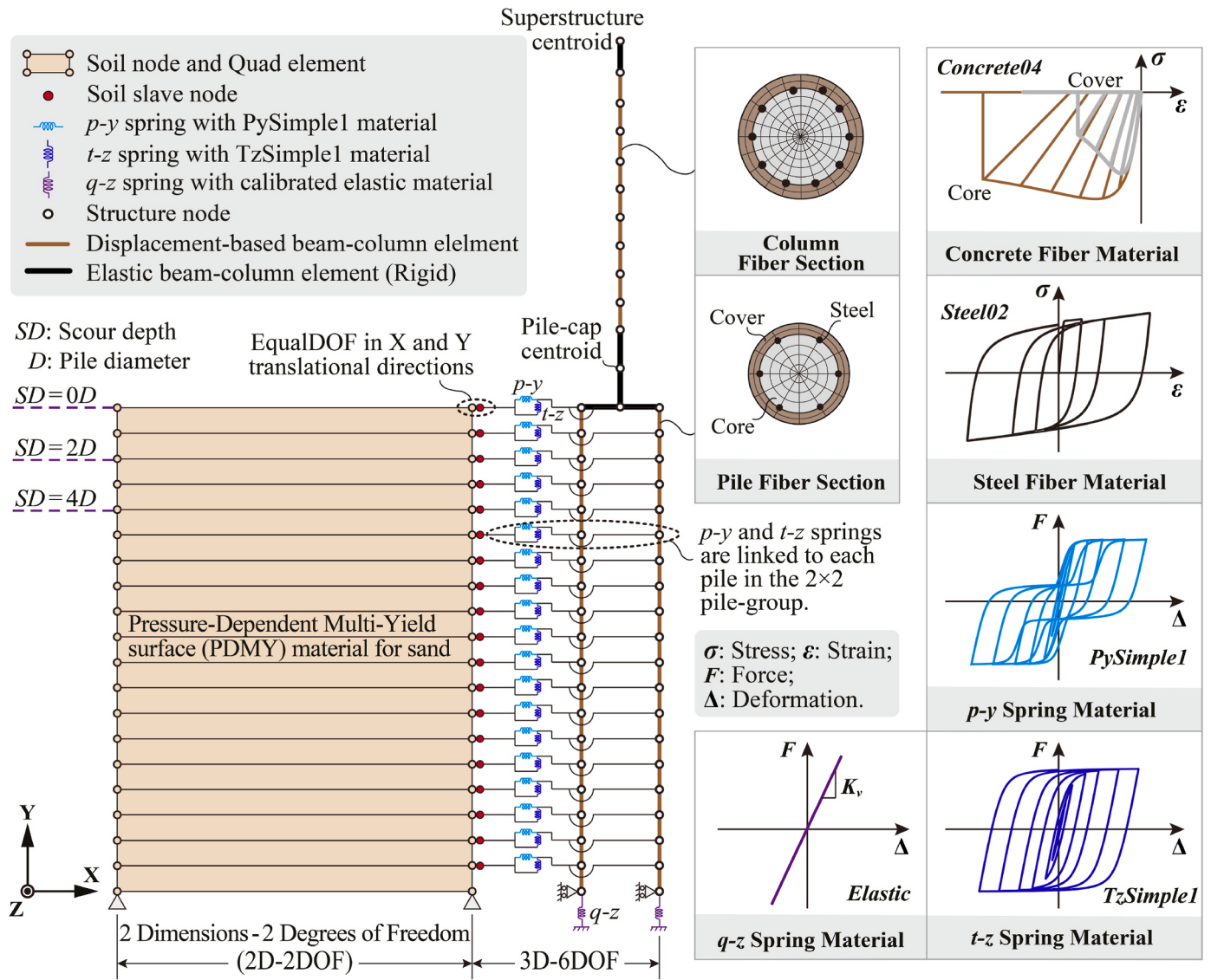


Fig. 11. Schematic illustration of coupled soil-pile-structure FE model for the SPSS shake-table test.

Table 2
Concrete material parameters for concrete fibers in the FE model.

Concrete type	Compressive strength (MPa)	Strain at peak strength	Ultimate strength (MPa)	Ultimate strain
Cover	32	0.002	6.4	0.005
Core	39.8	0.0037	7.2	0.010

Note: Compressive strength indicates the peak strength of concrete under compressive loading, while ultimate strength indicates the strength just before crushing (strength drop to zero).

Table 3
Steel material parameters for longitudinal steel fibers in the FE model.

Yield strength (MPa)	Elastic modulus (GPa)	Yielding strain	Post-yield hardening ratio	Ultimate strain
400	200	0.002	0.01	0.18

rial, especially the reference effective confining pressure (p'_r) and associated reference low-strain maximum shear modulus ($G_{r,max}$), are found paramount to capturing the soil column's predominant period and acceleration responses. More specifically, the *OpenSees* recommended value for $G_{r,max}$ of the test sand (i.e., around 75 MPa) under the recommended $p'_r = 80$ kPa is found not able to capture the predominant period and acceleration responses. A possible reason is that the low-strain maximum shear modulus of test soil at any given depth (G_{max}) was not well represented based on the recommended p'_r and $G_{r,max}$ through the well-known relationship (Yang, 2000):

$$G_{max} = G_{r,max} \cdot (p'/p'_r)^{0.5} \quad (1)$$

where p' is the mean effective confining pressure of soil, determined by:

$$p' = \frac{1 + 2 \cdot K_0}{3} \sigma'_{v0} \quad (2)$$

where $K_0 = 1 - \sin \phi = 0.5$ is the lateral pressure ratio with respect to the vertical counterpart for the test sand (Das, 2011); and σ'_{v0} is the vertical effective pressure. From Eq. (2), $p' = 1$ kPa for the surface element at a depth of $h = 0.1$ m (i.e., $p' = 0.66 \cdot \sigma'_{v0} = 0.66 \cdot \rho \cdot g \cdot h$, where $\rho = 1.53$ ton/m³ and $g = 9.81$ m/s² is the gravity acceleration constant), $p' = 4$

kPa at $h = 0.4$ m, and $p' = 19$ kPa for the bottom element at $h = 1.9$ m, resulting in $G_{\max} \approx 8, 17$, and 37 MPa, respectively, from Eq. (1). These $G_{\max}$ results are not the actual shear modulus of the test soils at the corresponding depths, as indicatively interpreted below.

To identify the actual $G_{\max}$ at $h = 0.4$ m, for instance, soil shear stress (τ) and strain (γ) responses at $h = 0.4$ m in the $SD = 0D$ case under the 2 Hz sinusoidal motion are derived and processed following three steps shown in Fig. 12. Firstly, soil acceleration and displacement records at $h = 0, 0.4$ and 0.8 m (i.e., Accelerometers A4 to A6 and Displacement Gauges D4 to D6, see Fig. 2) are used to derive τ and γ at $h = 0.4$ m as per (Zeghal et al., 1995). Secondly, τ - γ curves can be accordingly obtained as seen in Fig. 12(b), from which the low-strain maximum shear modulus is hard to estimate whereas the secant modulus at the peak strain is readily identified as 600 kPa. In the third step (Fig. 12(c)), the shear modulus degradation ($G/G_{\max}$) curve under $p' = 4$ kPa ($h = 0.4$ m) (Stokoe et al., 1999; Darendeli, 2001) is used to obtain the actual $G_{\max} = 5000$ kPa (i.e., 5 MPa). Note that different $G/G_{\max}$ curves under different p' from classical references (Seed and Idriss, 1970; Stokoe et al., 1999) are also plotted to demonstrate the adopted $G/G_{\max}$ curve for $G_{\max}$ estimate. Apparently, the actual $G_{\max} = 5$ MPa is less than one third of the recommended one (17 MPa). Therefore, $G_{r,\max}$ and p'_r in the PDMY material are set as those at $h = 4$ m (i.e., $G_{r,\max} = 5$ MPa and $p'_r = 4$ kPa), while soil materials at other depths are automatically determined in *OpenSees* via Eq. (1). Table 4 summarizes the adopted values and associated determination processes for the PDMY material parameters of interest for dry sands in this study, while the contraction, dilation, and liquefaction coefficients that control the change of pore pressure for below-water sands follow the *OpenSees* recommendations (Yang, 2000) and are not listed for conciseness. It is worth noting that different from the above modification approach for $G_{r,\max}$ and p'_r , another possible solution is to modify the exponent value (currently 0.5) in Eq. (1), which is out the scope of this paper, and will be explored in future studies.

4.4. Soil-pile spring modeling

4.4.1. p - y spring

Soil-pile springs represented by zero-length elements are modeled to simulate the seismic resistance of soils to the pile-group foundation. The horizontal soil resistance (p) at a depth (h) is represented by p - y springs with the PySimple1 material in *OpenSees* (Boulanger et al., 1999), which can be approximated by the American Petroleum Institute (API) p - y relation for sand (API, 2011):

$$p = A \cdot p_u \cdot \tanh\left(\frac{n_h \cdot h}{A \cdot p_u} \cdot y\right) \quad (3)$$

where y = soil-pile horizontal relative displacement; A = loading

Table 4
Adopted values for primary parameters in the PDMY material.

Parameter (Unit)	Value	Description
$^a D_r$	0.5	Relative density of the prepared sand deposit
ρ (ton/m ³)	1.53	Density of the prepared sand deposit
$^a \rho_{\min}$ (ton/m ³)	1.43	Minimum dry density of the test sand
$^a \rho_{\max}$ (ton/m ³)	1.65	Maximum dry density of the test sand
$^a G_s$	2.65	Empirical specific density of the test sand ^b
$^a e_{\min}$	0.52	Minimum void ratio, $e_{\min} = (\rho_w \cdot G_s / \rho_{\max}) - 1$, water density $\rho_w = 1.0$ ton/m ³
$^a e_{\max}$	0.85	Maximum void ratio, $e_{\max} = (\rho_w \cdot G_s / \rho_{\min}) - 1$
e	0.69	Void ratio, $e = e_{\max} - D_r \cdot (e_{\max} - e_{\min})$
φ (degree)	31	Frictional angle of the test sand measured from material tests
p'_r (kPa)	4	Reference effective confining pressure (depth of 0.4 m of the test)
$G_{r,\max}$ (MPa)	5	Reference low-strain maximum shear modulus estimated from the test
$^a K_0$	0.5	Lateral pressure ratio of the test sand, $K_0 = 1 - \sin \varphi$ ^b
$^a \theta$	0.33	Poisson's ratio of the test sand, $\theta = K_0 / (1 + K_0)$ ^b
$^a B/G$	2.67	Bulk modulus to shear modulus ratio, $B/G = [2(1 + \theta)] / [3(1 - 2\theta)]$ ^c
B_r (MPa)	13.4	Reference bulk modulus, $B_r = (B/G) \cdot G_{r,\max}$ ^c
$\gamma_{\max}$	0.1	Maximum shear strain ^c
d_p	0.5	Pressure-dependent coefficient, i.e., the exponent value in Eq. (1) ^c

^a Used to determine the parameters in the PDMY material model;

^b Das (2011);

^c Yang (2000).

condition coefficient, taken as 0.9 for cyclic ones like earthquakes; n_h = lateral resistance modulus depending on φ . Former in-site and laboratory tests have demonstrated the overestimate of lateral resistance modulus in the API code (Chai and Hutchinson, 2002; Wang et al., 2016, 2018b; Liu et al., 2020; Zhou et al., 2023). A possible reason is that the API code mainly serves for large-diameter stiff piles under working loads that trigger relatively low strains and consequently large lateral resistance moduli in soils as compared with small-diameter flexible piles under earthquakes. Therefore, relative small n_h values from ATC-32 (1996) are adopted, as shown in Fig. 13(a). Recall Eq. (3), p_u = ultimate resistance determined by (API, 2011):

$$p_u = \min \left\{ \begin{array}{l} (C_1 \cdot h + C_2 \cdot D) \cdot \rho \cdot g \cdot h \\ C_3 \cdot D \cdot \rho \cdot g \cdot h \end{array} \right. \quad (4)$$

where C_1 , C_2 and C_3 = coefficients for determining p_u , taken from Fig. 13 (b). Note that $\rho \cdot g$ in Eq. (4) represents the effective unit weight of dry sands in the shake-table test, and the measured density $\rho = 1.533$ ton/m³ is adopted, hereinafter inclusive. In addition, a pile-group efficiency factor of $\zeta = 0.8$ is taken as a multiplier to reduce p_u for the pile-group

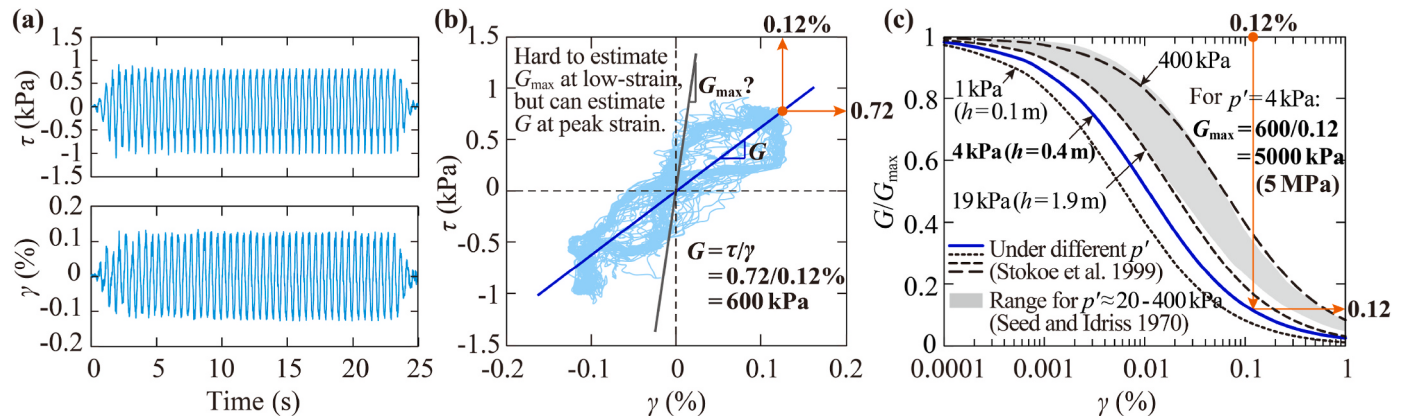


Fig. 12. Sand shear stress (τ) and strain (γ) responses at a depth of $h = 0.4$ m for $SD = 0D$ under the 2 Hz sinusoidal motion and estimation of maximum shear modulus ($G_{\max}$): (a) τ and γ time histories, (b) τ - γ relationship, and (c) sand $G/G_{\max}$ curves under different mean effective confining pressure (p') for estimating $G_{\max}$.

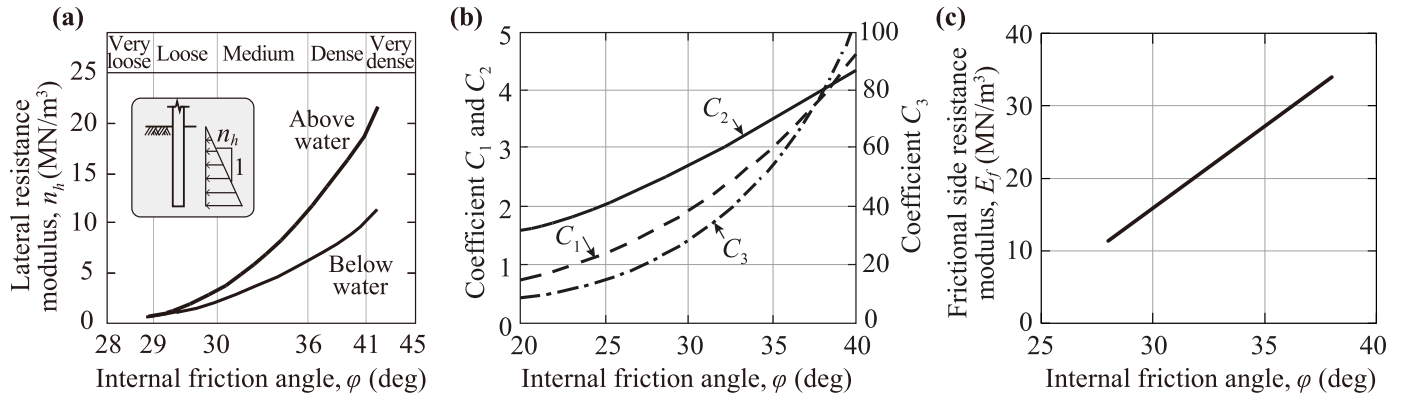


Fig. 13. Determination of values for key parameters in p - y and t - z spring materials for sand: (a) lateral resistance modulus, n_h in p - y springs (data from [ATC-32](#) and [ATC, 1996](#)), (b) coefficients C_1 , C_2 and C_3 for ultimate strength calculation in p - y springs (data from [API, 2011](#)), and (c) frictional side resistance modulus in t - z springs (data from [Mosher, 1984](#)).

effect of the 3D center-to-center distance ([Mokwa, 1999](#); [Lemnitzer et al., 2010](#); [Wang et al., 2019a](#); [Liu et al., 2020](#)). Besides, the PySimple1 material in *OpenSees* requires a parameter called y_{50} , which represents the soil-pile horizontal relative displacement when a half of p_u is reached, as calculated by:

$$y_{50} = \frac{Ap_u}{2n_h z} \ln \left[\left(1 + \frac{0.5}{A} \right) / \left(1 - \frac{0.5}{A} \right) \right] \quad (5)$$

4.4.2. t - z spring

Frictional side resistance along the pile skin (t) is modeled by t - z relationships ([Mosher, 1984](#)):

$$t = \frac{z}{\frac{1}{E_f} + \frac{z}{t_u}} \quad (6)$$

where z = soil-pile vertical relative displacement; E_f = initial modulus of side resistance related to ϕ , as seen in [Fig. 13\(c\)](#); and t_u = ultimate side resistance ([Kulhawy, 1991](#)):

$$t_u = K_o \cdot (\rho \cdot g \cdot h) \cdot (\pi \cdot d) \cdot \tan \delta \quad (7)$$

where $K_o = 1 - \sin \phi$ is the coefficient of lateral earth pressure at rest; and $\delta = 0.8 \cdot \phi$ is the internal friction angle between soil and precast concrete pile ([Kulhawy, 1991](#)). The t - z relationships are modeled by the TzSimple1 material in *OpenSees* ([Boulanger et al., 2003](#)). Again, the TzSimple1 material requests an input parameter called z_{50} that refers to the soil-pile vertical relative displacement when a half of t_u is reached, as determined by [Eq. \(8\)](#):

$$z_{50} = t_u / E_f \quad (8)$$

4.4.3. q - z spring

Recalling [Fig. 1](#), the pile-tip boundary condition of the shake-table test is characterized by welding of the reinforcement cage to the steel plate via a pair of crossed rebars and then concreting together; the steel plate was mounted to the bottom of laminar box. Therefore, the traditional nonlinear QzSimple1 material for pile-tips standing on soils ([Boulanger et al., 2003](#)) is not used in this study. Instead, an elastic material with a calibrated stiffness, $E_q = 40$ MN/m is assigned to the q - z springs at pile-tips. Note that E_q is calibrated by trying different E_q values from 20 MN/m to 100 MN/m with a step of 5 MN/m to ensure that the predicted fundamental period of the structure agrees with the recorded counterpart. Besides, it is worth noting that the elastic q - z spring is essentially similar to a pinned connection with a vertical resistance stiffness the same to the calibrated E_q . [Table 5](#) summarizes the adopted values of the key parameters in p - y , t - z , and q - z springs.

Table 5

Summary of adopted values for key parameters in soil-pile springs.

Spring	Material	Parameter (Unit)	Value	Description
p - y	PySimple1	A	0.9	Loading condition coefficient
		$C_1, C_2, \text{ and } C_3$	2.0, 2.8, 32	Coefficients for ultimate resistance calculation
		n_h (MN/m³)	4	Lateral resistance modulus of sand
t - z	TzSimple1	ζ	0.8	Pile-group efficiency factor
		E_f (MN/m³)	18.9	Frictional side resistance modulus of sand
q - z	Elastic	E_q (MN/m)	40	Calibrated pile-tip vertical stiffness

4.5. Motion input, damping, and solution scheme

For numerical model validation, the fixed soil nodes at bottom and the fixed ends of q - z springs (see [Fig. 11](#)) are imposed uniform excitations of the 0.1 g Victoria motion and the 0.1 g Chi-chi motion for the cases of $SD = 0D$ and $4D$, respectively. A low level of stiffness proportional damping coefficient of 0.006 is employed in light of laboratory tests on sand shear stress-strain hysteretic curves ([Kramer, 1996](#)). Krylov-Newton algorithm ([Scott and Fenves, 2010](#)), β -Newmark integrator with parameters of 0.3025 and 0.6, and a convergence tolerance of 10^{-4} on displacement residuals are used to solve the matrix equations of the FE model as numerically stable as possible.

4.6. Comparison between FE model prediction and test records

4.6.1. Vibration periods

From eigen analyses, the predicted first three vibration modes and periods (T_1 , T_2 and T_3) of the SPSS model with $SD = 0D$ are shown in [Fig. 14](#). It is evident that the first SPSS mode is dominated by the first-order vibration mode of column, while the second and third SPSS modes are dominated by the first- and second-order vibration modes of the soil column, respectively, where pile-structure vibrations are also mobilized. Besides, the FE model generally well captures the periods, as demonstrated by the accurately predicted T_1 and T_2 , although T_3 shows a disparity (i.e., 0.05 s versus 0.03 s). As for the case of $SD = 4D$, similar mode shapes are obtained, thereby not plotted for conciseness; the predicted first three vibration periods are 0.49s, 0.13s, and 0.05s, which are overall close to the recorded counterparts (i.e., 0.48 s, 0.10s, and 0.05 s).

4.6.2. Acceleration responses

Predicted and recorded acceleration responses for the cases of $SD =$

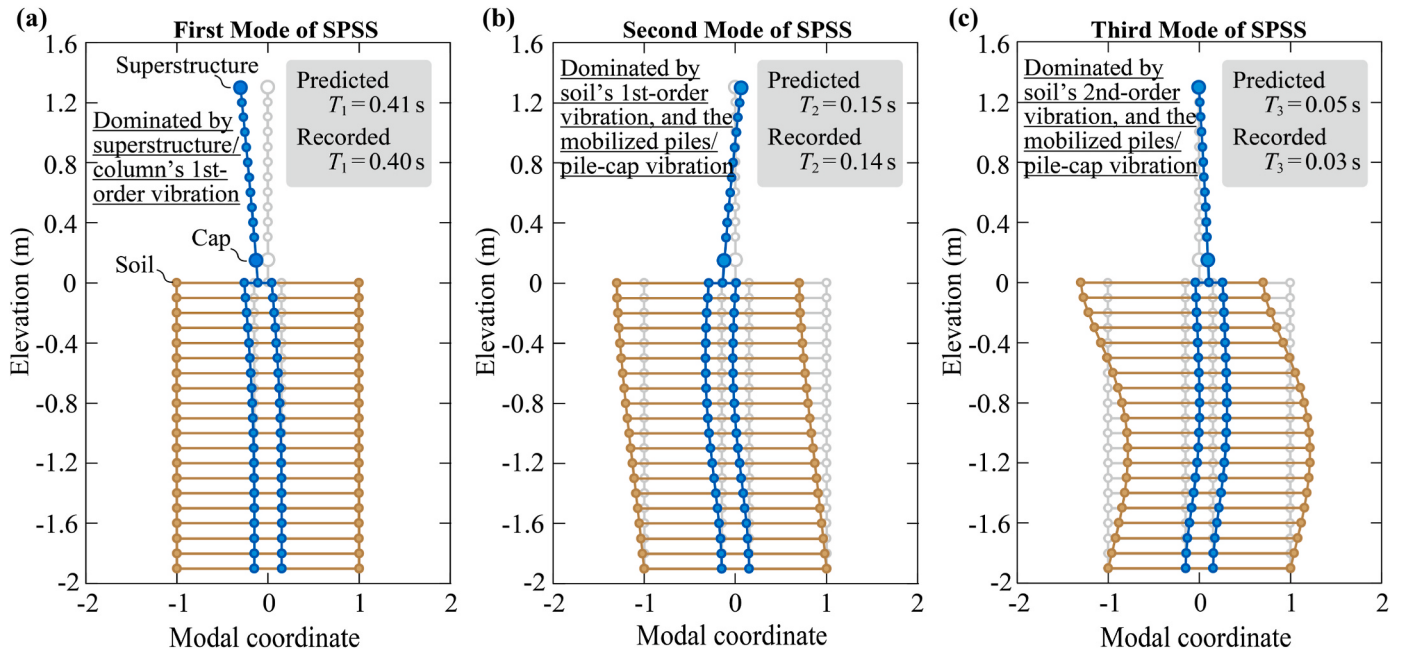


Fig. 14. Predicted vibration modes and periods, and comparison with recorded periods for the case of $SD = 0D$: (a) first mode, (b) second model, and third mode.

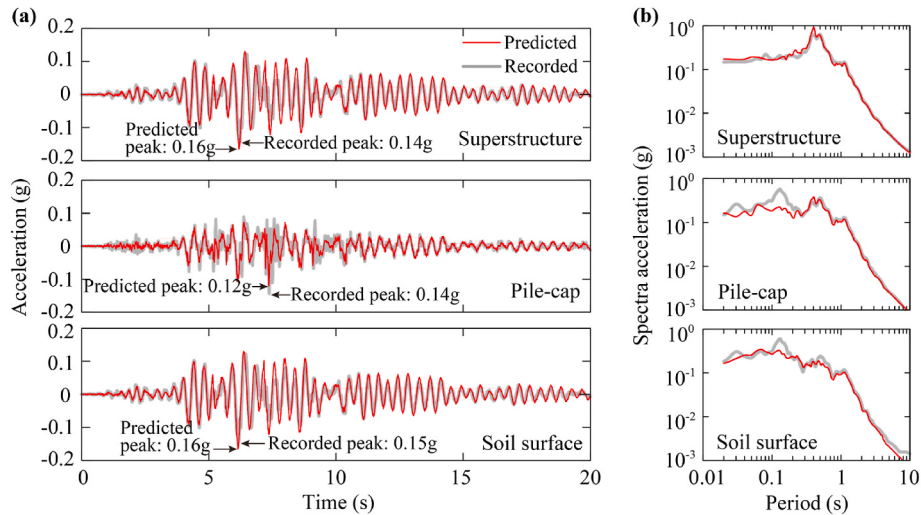


Fig. 15. Predicted and recorded acceleration responses at superstructure, pile-cap, and soil surface for the case of $SD = 0D$ under the 0.1 g Victoria motion: (a) time histories, and (b) corresponding response spectra.

$0D$ under the 0.1 g Victoria motion and $SD = 4D$ under the 0.1 g Chi-chi motion are compared in Fig. 15 and 16, respectively, in terms of accelerograms and associated response spectra. From Fig. 15(a), the time history phases and peak responses at the soil surface, pile-cap, and superstructure are very well predicted, with percentage differences of less than 15%. From Fig. 15(b), the acceleration response spectrum at the superstructure is perfectly captured across the period of interest, while those at the pile-cap and soil surface show some minor dispersions at relatively small periods, which may be partially attributed to the abovementioned disparity in the third SPSS mode period (recall Fig. 14 (c)). As for the case of $SD = 4D$ under the 0.1 g Chi-chi motion shown in Fig. 16, a similar level of prediction accuracy is observed. The superstructure acceleration is very well captured, while those on the pile-cap and soil surface exhibit some disparities at small periods, for which a potential reason is the difference in the predicted and recorded periods of the second mode (0.13 s versus 0.10 s) that is more related to the pile-cap and soil surface responses. Nevertheless, in short, the FE model can

reasonably capture the acceleration responses of SPSS.

4.6.3. Curvature responses and flexural behavior

To further understand the capability of FE model on the seismic curvature prediction of SPSS with different scour effects, Fig. 17 shows predicted and recorded curvature distributions at the epoch of maximum underground pile curvature under the 0.1 g Victoria motion for cases of $SD = 0D$, $2D$ and $4D$. From the predicted curvature distributions, it is clear that the location of the point of inflection shifts downward with the increasing scour depths, due to the reduction of embedded length for the piles. This trend generally conforms with the recorded counterpart. Besides, there is generally a better match at the column for a smaller SD , and a better match at the piles for a larger SD . The disparities may be attributed to the ignorance of (1) interaction between the cap bottom and the soil surface for the case of $SD = 0D$ and (2) accumulative invisible damage to the column (e.g., rebar low-cycle fatigue) triggered by the multiple sequential shakings from $SD = 0D$ to

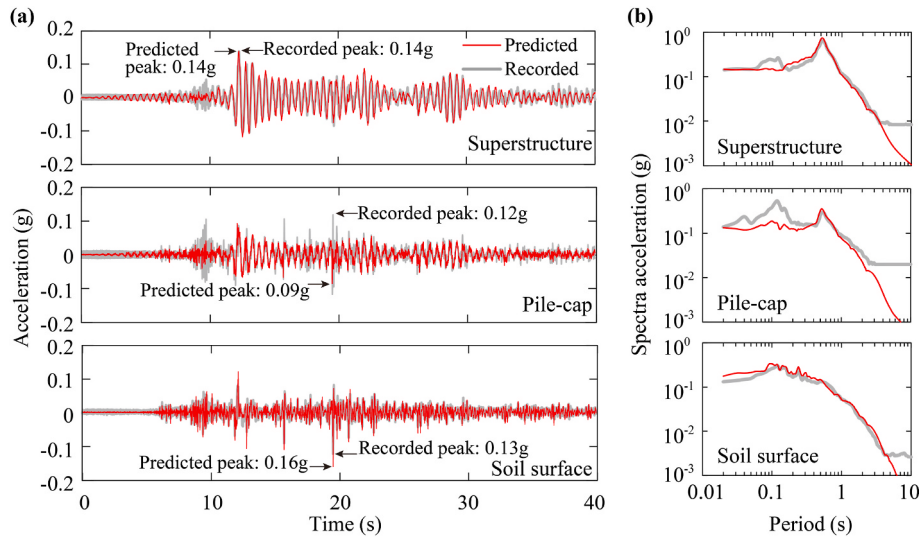


Fig. 16. Predicted and recorded acceleration responses at superstructure, pile-cap, and soil surface for the case of $SD = 4D$ under the 0.1 g Chi-chi motion: (a) time histories, and (b) corresponding response spectra.

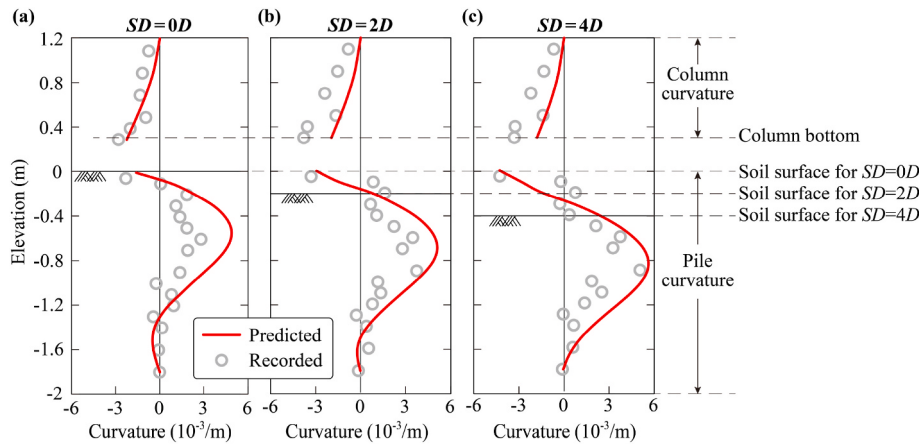


Fig. 17. Predicted and recorded curvature distribution at the maximum response epoch under the 0.1 g Victoria motion for different cases: (a) $SD = 0D$, (b) $SD = 2D$, and (c) $SD = 4D$.

4D even against quite a low intensity of input. Nevertheless, the developed FE model is capable of capturing seismic behavior of SPSS under scour.

5. Parametric analyses for quantifying resonance periods

5.1. Examined parameters

To identify the resonance periods of SPSS model with scour effects, parametric studies are conducted considering three parameters:

- (1) $SD = 0D$ to $6D$ with an interval of $1D$ that covers the potential scour scenarios in SPSS (Alipour et al., 2013). Note that the scour depth is expressed in the dimensionless form (times of D) to mitigate the limitation of the adopted non-unity scaling-factor in length (recall Table 1).
- (2) Sinusoidal wave frequency, $f_s = 0.5$ Hz–5 Hz with an interval of 0.1 Hz for 0.5–2 Hz, 0.2 Hz for 2–3 Hz, and 0.5 Hz for 3–5 Hz. Accordingly, the excitation period, $T_s = 1/f_s$ is determined ranging from 0.2 s to 2 s in the model scale, corresponding to a prototype range of approximately 0.6 s–6 s (recall Table 1), which generally covers the predominant periods of earthquake-prone

sites (Rathje et al., 2004). Note that a dimensionless parameter, the ratio of T_s to the SPSS fundamental period, T_1 , is defined as the **excitation period ratio**, T_s/T_1 , for mitigating the limitation of non-unity scaling-factor in period (recall Table 1) as well as yielding generalized conclusions. Note that for convenience of application, T_1 is defined as the elastic fundamental period of SPSS that can be obtained from eigen analyses of FE models, rather than the elongated fundamental period due to the effect of soil and structural nonlinearity under shaking.

- (3) Sinusoidal wave acceleration intensity from 0.1 g to 0.3 g with an interval of 0.1 g to understand the influence of structural damage extents (from elastic to nonlinear states) on resonance periods. Such an intensity range covers most of the design peak ground accelerations in a 475-year return period across China mainland (Standardization Administration of China, 2015), as well as most of the design peak ground accelerations in a 975-year return period across US (AASHTO, 2011). Note that as fundamental research, this study uses sinusoidal waves as input, rather than ground motions, to explicitly quantify resonance periods for SPSS with different scour effects.

5.2. Resonance period quantification

Curvature ductility demands at the bottom of column and the top of pile, computed by the maximum curvatures divided by yielding curvatures of individual sections, are taken as demand parameters to assess resonance excitation periods. Note that the yielding curvature of each section is determined by tracing the longitudinal rebar strain, i.e., when the yielding strain of 0.002 (recall Table 2) is reached. As such, variations of axial loads in piles under horizontal excitations are considered in determining the yielding curvature. The reason to assess the curvature ductility, rather than curvature, is two-fold. First, curvature ductility demands can reflect the elastic/inelastic behavior of structures (i.e., a ductility exceeding 1.0 indicates the inelastic behavior). Second, the curvature ductility demand is a dimensionless parameter that can mitigate the limitation of the non-unity length-scale-factor (i.e., curvatures from the numerical model need to be divided by the adopted length-scaling-factor of 10 to obtain the corresponding prototype results).

Based on nonlinear dynamic analyses, Fig. 18 and 19 displays three-dimensional relationships of the assessed two curvature demand parameters to the excitation period ratio and scour depth under three examined excitation intensities. For each scour depth, a noticeable peak demand that reflects the SPSS resonance behavior is observed at a specific excitation period ratio, T_s/T_1 , between 1 and 2. Such a specific excitation period ratio is defined to be called **resonance period ratio**, T_R/T_1 , where T_R is the resonance period that T_R/T_1 corresponds to the maximum curvature ductility demand. It is worth again noting that T_R/T_1 is a dimensionless parameter, which is rarely affected by the adopted non-unity scaling-factor of period (recall Table 1); thereby it can provide

useful insight for prototype structures.

More specifically, from the projection on the peak demand- SD relationships in Figs. 18 and 19, the column-bottom curvature ductility demands decrease with the increasing SD (i.e., being elastic under the 0.1 g excitation, changing from nonlinear to elastic states under the 0.2 g excitation, and being nonlinear under the 0.3 g excitation), whereas the pile-head demands, on the contrary, increase with the increasing SD (i.e., a trend from elastic to nonlinear states for the 0.1 g excitation, while being nonlinear for other excitation intensities). This phenomenon demonstrates the finding in literature that scour tends to transfer damage from column to pile (Wang et al., 2014, 2020b). Besides, from the projection on T_s/T_1 and SD , the resonance period ratio, T_R/T_1 is found generally increase with SD and the excitation intensity.

Someone may be interested in whether the soils behave nonlinearly in the parametric analyses. To address this concern, Fig. 20 showcases the normalized p - y responses (p and y responses divided by p_u and y_{50} , respectively) at an example depth of 0.4 m for cases undergoing different scour depths and excitation intensities at the identified resonance period ratios. Apparently, most cases show plump p - y hysteretic curves, particularly at the relatively large scour depths and excitation intensities, indicating the coverage of soil elastic to inelastic behavior in the parametric analyses.

To better quantify the resonance periods of SPSS, Fig. 21 gathers the above-identified resonance period ratio, T_R/T_1 in terms of the peak input acceleration and SD . It is found from Fig. 21 that T_R/T_1 are all between 1 and 2 for the examined cases. Recalling the column and pile curvature ductility demands versus scour depths, excitation periods and intensities (Figs. 18–19), it can be concluded that the variation of T_R/T_1 is primarily attributed to the yielding of the column and piles; the higher the

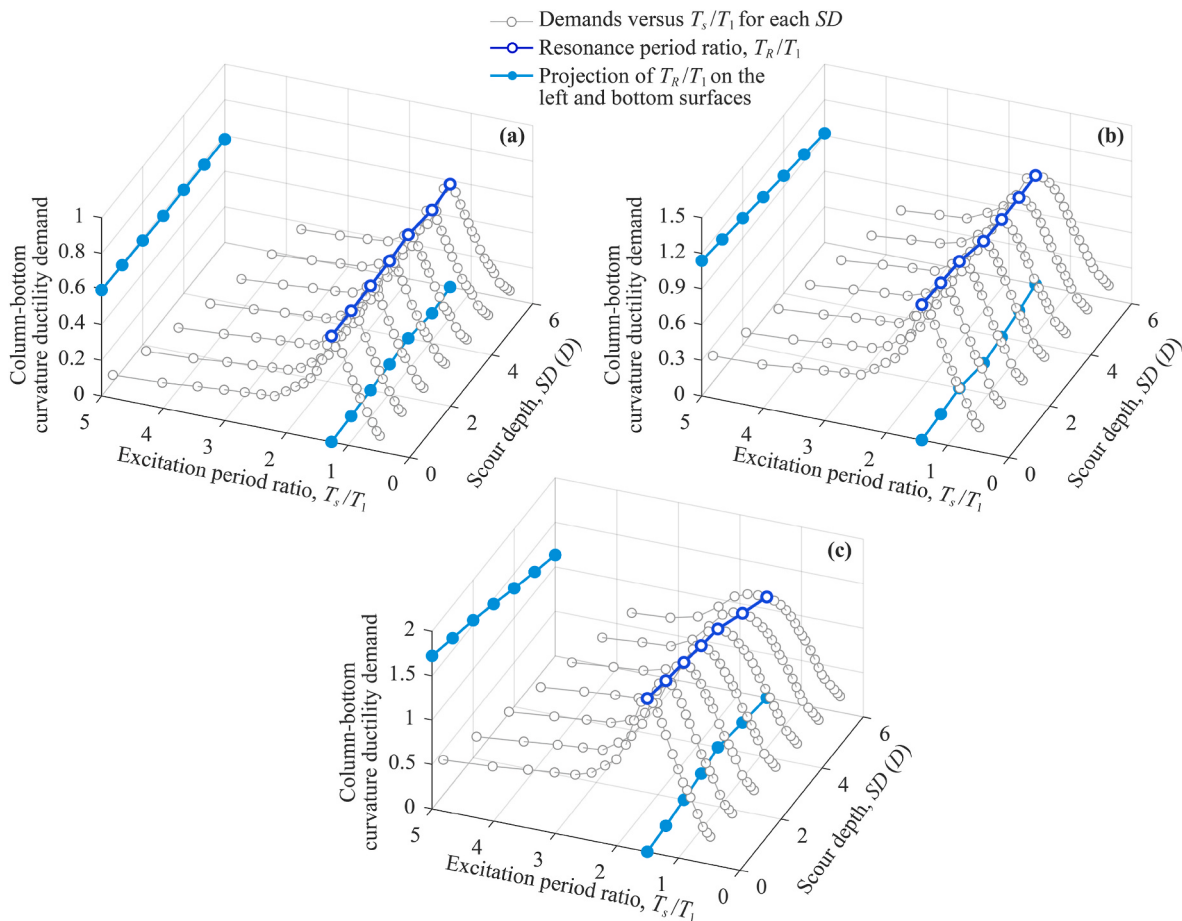


Fig. 18. Variations of column-bottom curvature ductility demands with respect to excitation period ratio (T_s/T_1) for different scour depths under different acceleration intensities: (a) 0.1 g, (b) 0.2 g, and (c) 0.3 g.

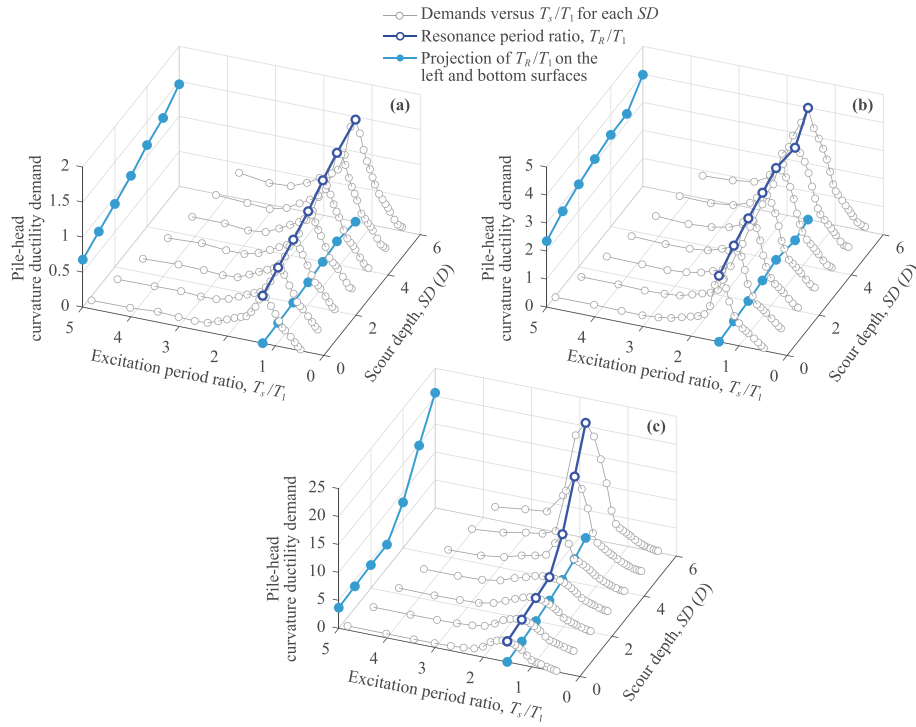


Fig. 19. Variations of pile-head curvature ductility demands with respect to excitation period ratio (T_s/T_1) for different scour depths under different acceleration intensities: (a) 0.1 g, (b) 0.2 g, and (c) 0.3 g.

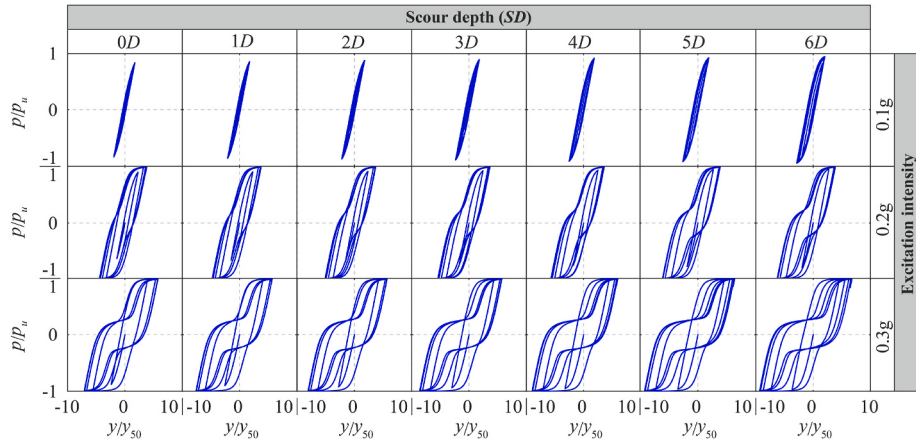


Fig. 20. Normalized p - y relationships at the depth of 0.4 m for cases undergoing different scour depths and excitation intensities at the identified resonance period ratios.

structural inelastic extent, the larger resonance period ratio (T_R/T_1).

For seismic analysis and design of SPSS, such resonance period ratios may be crucial and should be paid special attention for some specific scenarios. Considering the limitation and nature of sinusoidal waves applied in this study, a potential implementation of the above findings is for the seismic design of SPSS against narrow-band ground motions that are characterized by concentrated excitation periods (Somerville, 2005; Arroyo and Ordaz, 2007; Bojórquez and Ruiz-García, 2013). As such, SPSS undergoing scour potential shall be carefully designed to have proper fundamental periods, such that the adverse resonance period ratios between 1 and 2 can be avoided for the sake of structural seismic safety. For broad-band ground motions, on the other hand, the resonance issue may be not as critical as the narrow-band ones. However, it may become crucial for near-fault pulse-like strong ground motions (even broad-band) when the pulse period of the motion is close to the nature period of SPSS, i.e., special care should be taken to the pulse-like

strong ground motion that has a pulse period between 1 and 2 times the elastic fundamental period of the SPSS.

6. Conclusions

This paper investigates the seismic resonance behavior of the soil-pile-structure system (SPSS) considering scour effects. A series of shake table tests were conducted on an RC single pier structure supported by a 2×2 pile group embedded in cohesionless soils against white noise, harmonic, and historic earthquake excitations to study the dynamic characteristics and seismic behavior of SPSS. An advanced multi-dimensional numerical model is built with modified soil constitutive parameters and validated by the test results. Based on this model, extensive parameter analyses are performed to quantify the resonance periods for column-bottom and pile-head curvature demands of SPSS with different scour depths (0–6 times of pile diameter, D) and different

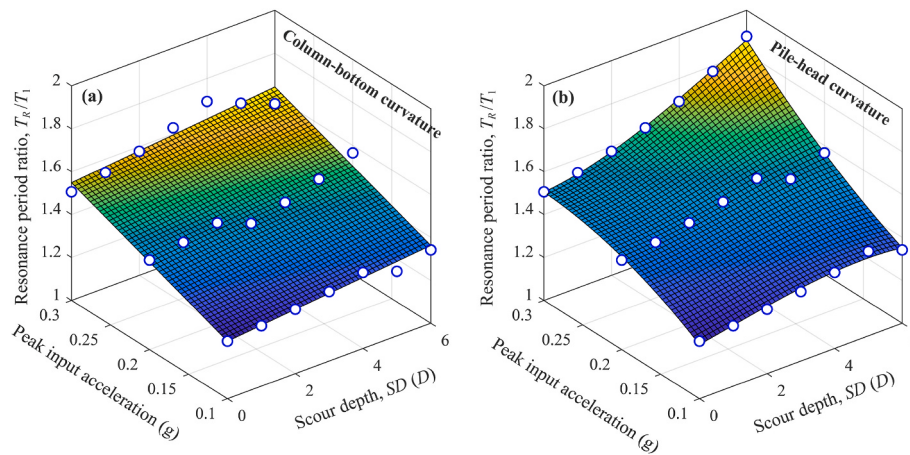


Fig. 21. Summarized and fitted surface for resonance period ratio (T_R/T_1) in terms of SD and peak input acceleration for different demand parameters: (a) column-bottom curvature ductility, and (b) pile-head curvature ductility.

excitation intensities (0.1 g–0.3 g). An excitation resonance period indicator named *resonance period ratio* (i.e., the period of the sinusoidal wave divided by the elastic fundamental period of SPSS) is defined in this study. The main findings are as follows:

- (1) Superstructure acceleration demands are dominated by the fundamental vibration mode of SPSS (deflection of the column and superstructure) where the superstructure inertial effect is the main contributor, while the pile-cap acceleration demands are mainly controlled by the soil vibration mode where soil-pile kinematic effects play an important part.
- (2) With column and pile behaving in the elastic state, the shake-table tests exhibit the resonance phenomenon that noticeably amplifies the structural curvature demands when the excitation period is close to the nature period of the SPSS model. With column and pile undergoing nonlinear behavior, the SPSS nature period increases, and the level of increase becomes higher when larger inelastic deformations are experienced. More importantly, the parametric study identifies an adverse resonance period ratio between 1 and 2, regardless of column or pile curvature ductility demand parameters across the examined scour depth and excitation cases. Moreover, the resonance period ratio mainly depends on the nonlinearity of SPSS; it generally increases with increasing scour depths and excitation intensities.

Although self-evident, it is still worth mentioning that the above findings are derived based on a series of scaled shake-table model tests and associated numerical analyses subjected to sinusoidal excitations. Nevertheless, since the findings on the adverse resonance periods are presented in the dimensionless form (i.e., resonance period ratio), they are theoretically rarely influenced by the adopted similitude law. The above findings can be preliminary documents for the advanced seismic design of SPSS prototypes with scour effects. Future studies will explore the resonance behavior of prototype structures in more complex soil profile contexts such as multi-layered cohesion and cohesionless soil sites.

CRedit authorship contribution statement

Xiaowei Wang: Methodology, Investigation, Visualization, Writing – original draft, Writing – review & editing, Funding acquisition. **Alice Alipour:** Methodology, Investigation, Writing – review & editing. **Jingcheng Wang:** Methodology, Investigation, Visualization. **Yu Shang:** Methodology, Investigation, Visualization, Writing – original draft. **Aijun Ye:** Conceptualization, Methodology, Supervision, Writing – review & editing, Funding acquisition.

Declaration of competing interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Data availability

Data will be made available on request.

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